

**THE EFFECT OF REDUCED BEAM SECTION
ON ELASTIC AND INELASTIC BEHAVIOUR
OF STEEL MOMENT FRAMES**

**M.Sc. Thesis by
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PREFACE

The purpose of this study is to investigate the effect of radius-cut reduced beam section on the elastic stiffness properties of steel framing elements. The results are then will be used to adapt this retrofitting scheme for future reference in Turkey.

The author wishes to thank his supervisor , Mr. Erdoğan UZGİDER ; who has provided valuable contributions to the study. Additional thanks are also for Mr. Özden ÇAĞLAYAN for his assistance as an experienced teammate.

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ABBREVIATIONS

RBS	: Reduced Beam Section
SMF	: Special Moment Frame
ASTM	: American Society for Testing of Materials
SMRF	: Special Moment Resisting Frame
M	: Magnitude (of an earthquake)
AISC	: American Institute of Steel Construction
AWS	: American Welding Society
OMF	: Ordinary Moment Frame
FEMA	: Federal Emergency Management Agency
CJP	: Complete Joint Penetration
RBS	: Reduced Beam Section

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LIST OF SYMBOLS

σ	: Stress
σ_y	: Yield stress
E	: Young's modulus
J	: Torsional constant
E_{sh}	: Strain-hardening modulus
L	: Span length
I_i	: Moment of inertia with respect to i axis
k_j	: Modification factor for the property j
ϵ	: Strain
ϵ_{ult}	: Ultimate strain
δ	: Displacement

ZAYIFLATILMIŞ KİRİŞ KESİTİNİN MOMENT AKTARAN ÇELİK ÇERÇEVELERİN ELASTİK VE ELASTİK OLMAYAN DAVRANIŞINA ETKİSİ

ÖZET

1994 yılında meydana gelen Northridge depreminin ardından, o güne dek moment aktaran çelik çerçevelerde yaygın olarak kullanılan kaynaklı kiriş-kolon birleşimlerinin, deprem sırasında kendilerinden beklenen davranışı göstermedikleri ve tasarım aşamasında gözönüne alınan davranış düzeyine ulaşmadan göçme sınır durumuna eriştikleri gözlenmiştir. Bu sorunu çözmek amacıyla bir çok yaratıcı çözümler önerilmiştir. Bu tekniklerden biri olan zayıflatılmış kiriş kesiti çözümü ile, kiriş-kolon birleşimine yakın bir bölgede kiriş başlığında azaltma yapılarak taşıyıcı sisteme bir çeşit yapısal sigorta yerleştirilmektedir. Bu çözüm, kirişlerde birleşimden uzaktaki küçük bir bölgede enerji yutulmasını sağlamaktadır. Bu çalışmada, zayıflatılmış kiriş kesitinin taşıyıcı kiriş elemanının elastik rijitliğine ait karakteristik terimlere olan etkisi bir genel sonlu eleman analizi yazılımı yardımıyla hesaplanmıştır. Bulunan sonuçlar ve bu sonuçların olası kullanımları hakkında değerlendirmeler yapılmıştır.

THE EFFECT OF REDUCED BEAM SECTION ON ELASTIC AND INELASTIC BEHAVIOUR OF STEEL MOMENT FRAMES

SUMMARY

After the 1994 Northridge earthquake, it was observed that the welded beam-to-column connections could not withstand the expected earthquake distortions and had severe problems that led to limit states below the intended design levels. In order to solve this problem, many innovative techniques have been proposed. One of these techniques is the reduced beam section, so-called the dogbone , that introduces a structural fuse in the frame system by reducing beam flange width close to the connection. This scheme helps the frame dissipate energy in a stabilized manner in small portions of the beams, away from the connection. During this study, the effect of dogbone existence on stiffness characteristics of beams is computed with the help of a general finite element analysis software. The results are then reviewed for future reference.

1. INTRODUCTION

Among many other structural systems, steel frames have always been considered as the most ductile structural framing system by structural engineers as well as academicians and building authorities. This idea has evolved from the ductile material property of structural steel, which helps the building to undergo inelastic deformations easily during an earthquake, and in this way dissipate considerable amount of seismic input energy. Knowing that structural steel is an efficient solution for earthquake resistant building design problem, research has been conducted for many years for developing better structural solutions, again with the use of structural steel. New techniques have evolved from these research campaigns, with many of them ending up with a proposal or regulation for a new way of design as an outcome. These special designs employ concentric as well as eccentric bracing elements or knee-brace elements and finally researchers have even tried and succeeded in using reinforced concrete shear walls with these specially braced steel frames to resist earthquake induced forces acting on the structures, particularly buildings.

As mentioned above, ductile behaviour of structural steel plays the key-role in response of these structures during an earthquake. However, it has been observed that excellent inelastic behaviour and energy dissipating capacity of this superior construction material does not guarantee that the structural steel frame as a whole is earthquake resistant. Engineers who had designed numerous steel moment resisting steel frames for many years, faced this situation after the great earthquake that hit Northridge (USA) in 1994 [1]. Common failures of the widely used moment resisting steel frames were observed within the post-earthquake investigations, many of them relating to the beam-to-column connections [2]. The common practice of at first field-bolting a shear tab which is shop-welded to the beam web to the column flange and then groove welding the end of the beam flanges to the column flange in the field was under suspect. After the earthquake, government authorities quickly established a research project involving many top-rank colleges and institutions in the United States to study and investigate the topic and find possible causes of this

widespread damage to the moment resisting steel frames [3]. The first part of this major research programme has dealt with the commonly used structural steel beam-to-column detail, known as the pre-Northridge connection [4]. Academic efforts within the universities were jointly supported by the experts of structural design working as consultants in respected engineering offices. The outcome of the first phase of this research regarding the investigation for possible causes of this damage led to a secondary research phase, dealing with providing new solutions for the common problems regarding the overall and local behaviour of steel frames, mainly connections [5]. The long-term study has recently ended up with new recommended seismic design criteria for steel moment buildings [3].

The cures for the brittle behaviour of welded steel moment frame connections vary from each other. Some researchers have provided solution strategies for the welds [6], whereas some research groups extensively dealt with modifying the connections, such as adding gusset plates or haunches. Among all, one retrofit scheme is considered to be very innovative. This scheme inserts a “structural fuse” into the steel framing system by reducing the flange width of a portion of the beam, at some distance away from the connection. The reduced beam section, also known as the “dogbone” , decreases the demand on the groove welds connecting the beam flanges to the column flange. The problem of the pre-Northridge type connection was overcome with this idea and tests that have been performed for understanding the behaviour of beam-to-column connections having reduced beam sections [7]. The results were very promising, with the specimens easily achieving plastic rotation demands that were stated in the design codes.

The idea of weakening the beam for strengthening the structural frame may easily be considered as nonsense. However, test results clearly demonstrate the superior performance of this type of beam-to-column connections. One may ask for the effect of this reduced portion of the beam on the global as well as local performance of frames. This scheme, in fact, reduces the overall stiffness of the structural system ; but this reduction guarantees the achievement of “strong column – weak beam” design philosophy.

In this study, the effect of reducing the beam flange width on the global behaviour of the frame element is investigated. The alteration in overall rigidity also alters the

lateral stiffness and other stiffness characteristics of the structural system. As a consequence, increase in deformations such as lateral drifts is unavoidable. This study deals with the effect of dogbone type framing system on behaviour of steel moment frames in terms of stiffness characteristics of the frame element.

For understanding the behaviour of many different systems, finite element analyses provide an excellent medium. Research groups have always used general finite element analysis softwares extensively, mainly to support their experimental parts of researches [8]. Knowing that experimental investigation is time-consuming and costly, these computer-based analyses clearly help us to predict the behaviour of various type of structural frame or beam-to-column configurations.

For this reason, the parametric investigations within this study have been conducted with the help of a commercial general finite element software. The results have then been reviewed.

2. STRUCTURAL STEEL

2.1 Overview

Despite the fact that only limited number of structural shapes is rolled in Turkey, the situation in foreign countries is somehow different. But similarly, for the first half of the 20th century, there was essentially only one type of structural steel widely available in North America and that was Grade A-7 steel. By the 1960s, engineers had to come face to face with different types of steel products, although only one or two of them were used in general. When we come to our time, most engineers mix two different grades of steel in structural applications.

Contrary to this amazing growth of steel products, the minimum requirements for structural steel grades have remained relatively simple. Limits on mechanical properties such as minimum yield stress and tensile strength, and minimum percentage elongation prior to failure have not changed so much. This wide range of structural steel products brings together the challenge for selecting proper grades for construction and detailing. Using different steel grades may cause improper and unexpected behaviour during the life of the structure.

Considering the above mentioned topics, a brief review of structural steel properties will be done. These subtitles do not cover all the important issues regarding the structural steel properties, but have the major importance in behaviour. These are the stress-strain relationship of structural steel, plasticity and hysteretic behaviour and strain rate effect on tensile and yield strengths.

2.2 Common Properties of Steel Materials

These important properties are the stress-strain relationship, which helps us define the grade of steel ; plasticity and hysteretic behaviour, which enables the structural steel framing to undergo inelastic deformations and in this way dissipate energy ; and

the strain rate effect on tensile and yield strengths ; which is again very important in behaviour of structural steel connections [9].

2.2.1 The stress-strain relationship

This relationship plays the key role in defining the structural steel grades. The relationship is shown in the figure below. The main important points in the figure are the upper and lower yield strengths ($\sigma_{y \text{ upper}}$ and $\sigma_{y \text{ lower}}$, consecutively) , and the elongation at the onset of yielding , shown by ϵ_y .

In steel rolling mills, many applications are applied on the steel material. These efforts mainly aim at achieving higher yield strengths by alloying or quenching and tempering, but all these efforts reduce the elongation up to failure and the length of the plastic plateau.

The typical structural steel stress-strain relationship can be seen in Figure 2.1 .

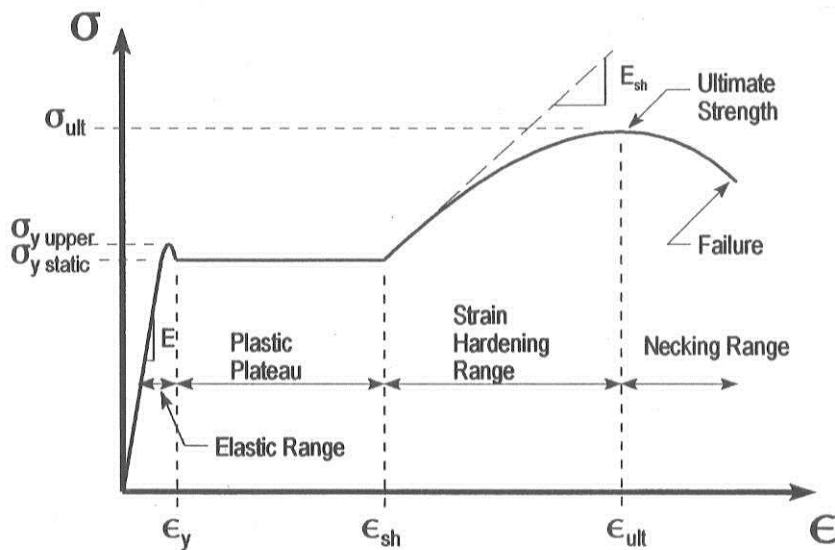


Figure 2.1 Stress-strain curve for structural steel.

In this figure, ϵ_y , ϵ_{sh} and ϵ_{ult} represent the strains at the onset of yielding, strain hardening and necking, respectively. For a typical structural steel material, the modulus of elasticity can be taken as 210000 MPa (N/mm²) and the tangent modulus at the onset of strain hardening can be taken as 1/30th of this value, as about 7000 MPa.

2.2.2 Plasticity and hysteretic behaviour

A very important property of structural steels is their superior ability to dissipate large amount of hysteretic energy when they are subjected to large cyclic inelastic loading. The energy that is needed to plastically deform a steel specimen can easily be calculated as the area under the force – displacement curve that is in the inelastic range which is obtained when the steel specimen is subjected to monotonically increasing load. This noncoverable energy is named as the hysteretic energy. The progressive increase in axial load which creates this hysteretic energy should be large enough to be able to deform the specimen into the inelastic range.

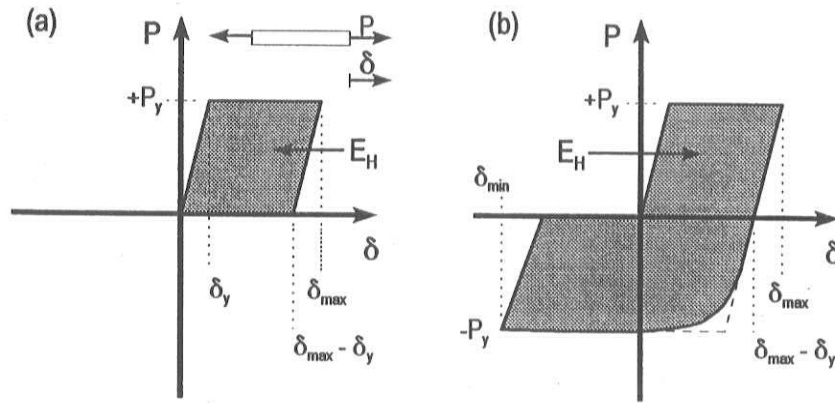


Figure 2.2 Hysteretic energy of structural steel

As shown in Figure 2.2 , this progressively increasing loading and unloading condition leads to the hysteretic energy , E_H , which is expressed as :

$$E_H = P_y (\delta_{MAX} - \delta_y) \quad (2.1)$$

which represents the shaded area in the figure.

When the steel specimen is loaded and unloaded cyclically, an amount of energy will be dissipated at each cycle. The summation of the absolute values of these energy dissipations lead to the total dissipated energy. This cumulative energy dissipation capacity is very important in earthquake resistant design, and careful detailing is necessary for ensuring this type of behaviour.

2.2.3 Strain rate effect on tensile and yield strengths

This appears to be another factor that affects the shape of the stress-strain curve. When loading is in a fast manner, the yield and tensile strength stresses will appear to be higher than expected. This situation results in extreme demands on important portions of the frames, mainly connections. A condition where strain rate is very high is blast effect, and the connections require special care when designing the structure as blast-resistant.

3. EARTHQUAKE FAILURES OF WELDED CONNECTIONS

3.1 Overview

Each year, major disasters involving engineered structures take a significant human and economic toll around the world. Among the principal responsibilities of the engineering community is to continually inspect the performance of these structures and to promote corresponding refinements in design methods, construction practices and material selection to better resist the next challenging event.

While continual refinement undoubtedly leads to a better understanding of structural dynamics and material behavior, and consequently tougher buildings, mistakes and failures can still be expected. The January 1994 earthquake in Northridge, California introduced the beginning of another such cycle, as building owners discovered that dozens of steel frame buildings, which they were told represented the state-of-the-art in aseismic design, suffered severe cracking [1,2]. In fact, prior to the Northridge earthquake, welded steel moment resistant frames enjoyed the full trust of construction communities in the world's most advanced seismic areas of Asia and California. These structures were assumed to be strong enough to resist the stresses, and ductile enough to accommodate the distortions generated by severe earthquakes. Instead, the connection between the beams and columns fractured at load and deformation demands well below those for which they were intended.

This chapter deals with the damage that was observed during the post-earthquake inspections which were carried out as a routine process, but which shocked the building authorities.

3.2 The Northridge Earthquake and Its Impacts

At 4:30 on the morning of January 17, 1994, about 10 million people in the Los Angeles region of southern California were awakened by the shaking of an

earthquake. The earthquake, named for its epicenter in the town of Northridge, was a magnitude 6.7 ($M = 6.7$) shock that proved to be the most costly earthquake in United States history. The shaking heavily damaged communities throughout the San Fernando Valley and Simi Valley, and their surrounding mountains north and west of Los Angeles, causing estimated losses of 20 billion dollars. Fifty-seven people died, more than 9,000 were injured, and more than 20,000 were displaced from their homes by the effects of the quake [1,2].



Figure 3.1 General layout of the earthquake location. [1]

Although moderate in size, the earthquake had immense impact on people and structures because it was centered directly beneath a heavily populated and built-up urban region. Thousands of buildings were significantly damaged, and more than 1,600 were later “red-tagged” as unsafe to enter. Another 7,300 buildings were restricted to limited entry (yellow-tagged), and many thousands of other structures incurred at least minor damage. The 10-20 seconds of strong shaking collapsed buildings, brought down freeway interchanges, and ruptured gas lines that exploded into fires. By chance, the early morning timing of the earthquake spared many lives that otherwise might have been lost in collapsed parking buildings and on failed freeway structures.

3.3 Steel Moment Resisting Frame History

The type of structure that is the subject of this paper is the steel moment resisting frame or SMRF. The frames are characterized by rigid connections between the beams and columns that force the entire frame to deform when subject to lateral load. In theory, under intense lateral loads resulting from seismic ground motion, energy should be dissipated in a stable manner as the frames distort. For severe seismic events, the beams would undergo substantial inelastic deformations, but the building should not collapse, thus, protecting the lives of occupants. The concept was popularized by the architectural desire to increase the space unobstructed by braces and shear walls in buildings, leading to steel frame construction with large open bays and correspondingly large structural members [10].

SMRFs have been used to resist lateral loads from wind and earthquakes since the turn of the century, when partial connection rigidity was accomplished through hot riveting, and later, bolting. Relatively recently, in an effort to ease the constructability of these structures (that is, lower the construction costs), a beam-to-column connection was developed based on full penetration welds between the beam and column flanges. Use of welded steel moment resisting frames in the construction of commercial buildings in seismically active regions has been a common practice since the early 1970s. SMRF technology is used around the world; it has been estimated that there are some 20,000 SMRF buildings on the west coast of the US and Canada alone [9].

Since the conception and original testing of welded SMRFs, several aspects of steel frame building design have changed. First, typical beam and column sizes in buildings have increased, and as such, fewer frames per building are needed to resist the lateral loading. Consequently, structural redundancy has been reduced. Secondly, flux-cored arc welding (wire) has essentially replaced shielded metal arc welding (stick) for all field welding of steel moment connections. The switch to wire changes the mechanism and rate by which the weld metal is deposited, at the same time delivering a substantially more brittle weld metal. The parent steel has also changed with time, as yield strengths have crept upward (particularly for steel delivered to meet minimum ASTM A36 requirements). All these changes were being introduced in an evolutionary effort to lower the cost of SMRF buildings. Because each change

was done in an evolutionary and fragmented manner, little additional laboratory testing was triggered to assess the individual changes.



Figure 3.2 Divot type connection crack with no plastic deformation.

While thousands of SMRF buildings were being constructed, limited testing of the connection and further refinement of the connection details continued. Although some problems were noted in the laboratory performance of the connections, most engineers and researchers continued to trust the connection would perform as intended. Most notable is the work done at the University of Texas, Austin in the early 1990s, where premature cracking of SMRF connections during tests was widely published. The engineering community met the implication that there would be widespread cracking of SMRF buildings in an earthquake with skepticism. Eventually, the real-world test by the Northridge earthquake demonstrated behavior that was markedly different from the design intent (Figure 3.2) : connection fracture with little or no gross plastic deformation.

Not surprisingly, the dramatic failures of seemingly ``earthquake-proof" buildings has caused much excitement in the engineering, construction and commercial building owner community. The federal government has provided millions of dollars

in research and testing of SMRF connections to determine what went wrong, what the best ways are to repair earthquake damaged SMRFs, and how best to design and build the next generation welded SMRF buildings that will perform reliably in the next earthquake [3]. After five years and tens of millions of dollars in research, little consensus has been reached among engineers regarding the root causes of the cracking and most appropriate remedies.

3.4 The Typical Connection

Virtually all west coast SMRF-buildings constructed in the last three decades have used the same standard connection detail [4,11] ; welded flange and bolted web (Figure 3.4). Until recently, all elements of the connection had been prescribed in detail by the building code, leaving little room for the individual engineer to deviate from the standard. It was, in effect, a code-dictated rigid connection detail for steel frames. Acceptance of the moment connection resulted from many years of testing and research, primarily at University of California, Berkeley. Like virtually all structural connection details, the standard SMRF connection contains local stress risers, such as reentrant corners and bolt holes.

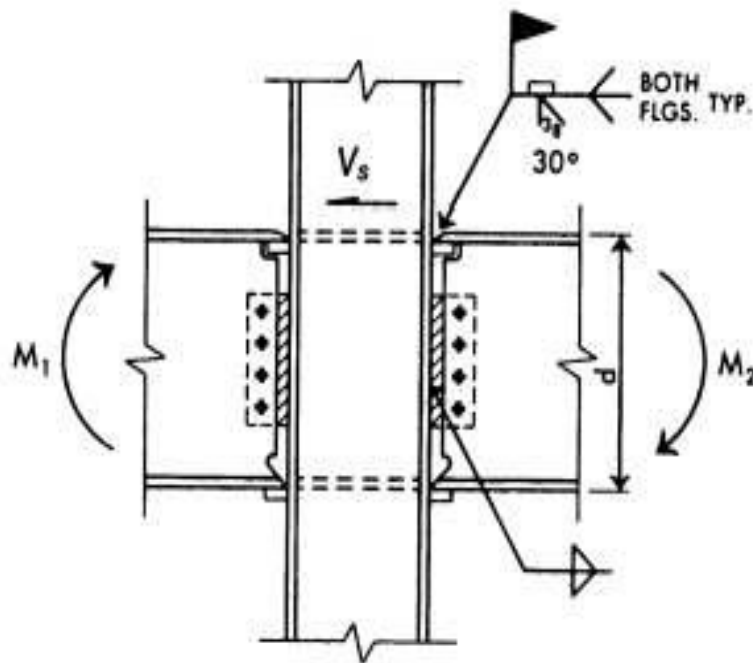


Figure 3.3 Typical pre-Northridge connection detail.

However, designers have generally assumed that localized material yielding will redistribute the stress and mitigate the effects of such concentrations. Through redistribution, the typical SMRF connection was expected not only to carry the full moment capacity of the beam, but also to deform in a ductile manner well past its elastic limit. The welded joint between the column and beam flange was simply expected to transfer the associated stresses, without particular attention to the stress riser associated with the full-penetration, beveled groove weld joint detail. Properly detailed and fabricated, this weld configuration is considered "prequalified" under the building code based on a perceived history of good performance. All contributed to the final form and application of the standard SMRF connection.

3.5 The Suspected Points Regarding the Poor Performance of Connections

The research to date has identified many technical factors that contribute to the SMRF connection damage [3]: standard use of weld materials that result in low notch toughness weld deposits; stress triaxiality due to connection geometry; high stress (strain) risers at geometric discontinuities; unreliable steel properties in the direction perpendicular to rolling; design practice that favors relatively few moment resisting frames; larger sections than had been previously tested; difficulties in non-destructive inspection of connections; excessively weak and flexible column panel zones; and highly variable and unpredictable material properties of beams and columns, particularly beam yield strength. In fact, the effort to understand this connection's behavior since its shortcomings were discovered in the Northridge earthquake is at least an order of magnitude greater than the total effort expended to develop and refine the design prior to 1994.

3.6 A Brief Review of Prequalified Post-Northridge Welded Connections

Prequalified connection details are permitted to be used for moment frame connections for the types of moment frames and ranges of the various design parameters indicated in the limits accompanying each prequalification. Project-specific testing should be performed to demonstrate the adequacy of connection details that are not listed as prequalified, or are used outside the range of parameters indicated in the prequalification.

Some important prequalified connection types and further information can readily be obtained from FEMA documents [3].

3.6.1 Reduced Beam Section Connections

This concept may be used for the design of fully restrained, Reduced Beam Section (RBS) connections. These connections utilize circular radius cuts in both top and bottom flanges of the beam to reduce the flange area over a length of the beam near the ends of the beam span (Figure 1) [12]. Welds of beam flanges to column are complete joint penetration groove welds, meeting the requirements of FEMA-353 (Recommended Specifications and Quality Assurance Guidelines for Steel Moment Frame Construction for Seismic Applications) [13]. In this type of connection, no reinforcement, other than weld metal, is used to join the flanges of the beam to the column. Web joints for these connections may be either complete penetration groove welds, or bolted or welded shear tabs.

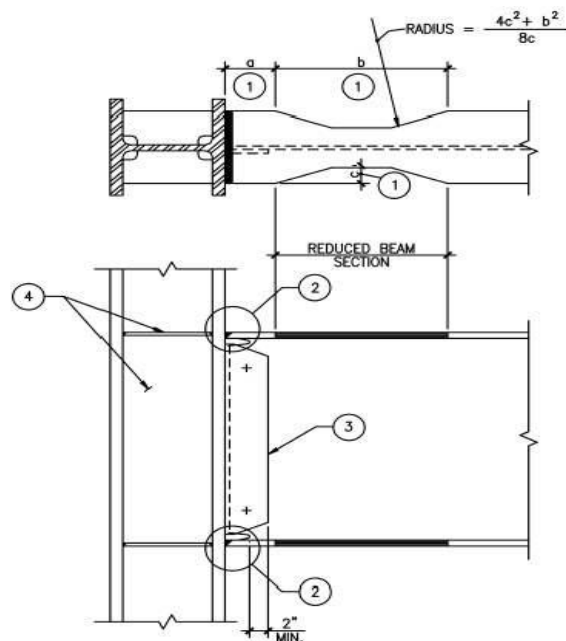


Figure 3.6 Reduced Beam Section Connection [3]

This type of connection has performed adequately in tests with both welded and bolted web connections. While a welded web connection is more costly than the more conventional bolted web connection, it is believed that the welded web improves the reliability of the connection somewhat. The welded web provides for

more effective force transfer through the web connection, thereby reducing stress levels at the beam flanges and beam flange groove welds.

3.7 Closure

This chapter was mainly devoted to introducing the experienced behaviour of steel structures during major earthquakes, particularly the Northridge earthquake. The aim of designing structural steel moment frames was given in a brief manner. While doing this, the commonly used pre-Northridge welded connection detail was examined and possible prequalified post-Northridge connection details were briefly presented. The following chapter deals mainly with the idea and design of reduced beam section, which is the main topic of this study.

4. THE REDUCED BEAM SECTION CONNECTION

4.1 Overview

When a major earthquake hits a structure, the intention is that, buildings which are designed to meet the design requirements of typical building codes are expected to have damage to both structural and nonstructural elements within the structure. However, the intensity of the damage has a great effect on the possibility of reusing the structure. The response of a structural system mainly depends on the ability of the structure to respond to the earthquake beyond its elastic range, therefore dissipate seismic input energy. The ductility is therefore very important, for it provides the structure to undergo inelastic deformations without any sudden loss of rigidity or stiffness [9].

In this chapter, the commonly used special moment frame (SMF) system is introduced and pre-Northridge connection detail is reviewed. A new and innovative type of steel moment frame beam-to-column connection, known as the “reduced beam section” or “dogbone” , is presented with its historical background.

4.2 Description of Special Moment Frames

The objective in earthquake resistant structural frame design is to dissipate energy in the moment frame beams and panel zones as indicated in Figure 4.1 below on the left-hand side. The design procedures are intended to avoid soft-story mechanisms such as that shown in the right-hand figure [9].

A SMF lateral force resisting system is often preferred by building owners and architects because this type of system provides large unobstructed spaces throughout the building plan. This "open" layout offers the most flexibility for programming the spaces as well as architectural appointments. For these reasons, steel buildings with SMF systems are quite common in major commercial and institutional structures.

Furthermore, the SMF system is considered by many to be one of the most ductile steel building systems available to the engineer. For this reason, SMF systems have been widely used in areas of high seismicity [10].

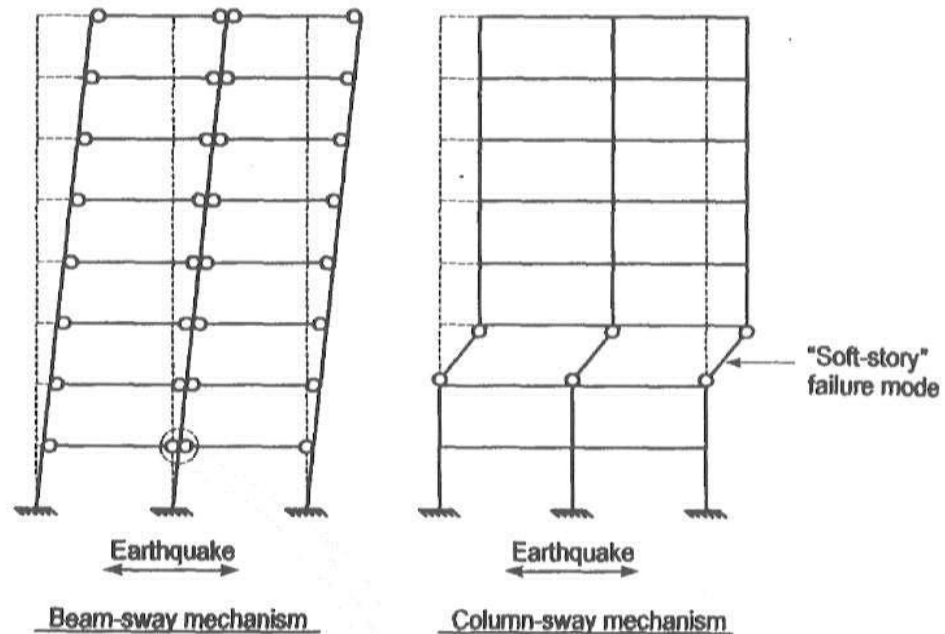


Figure 4.1 Possible frame mechanisms

SMFs are typically comprised of connections between wide flange beams and columns where beam flanges are welded to column flanges utilizing complete joint penetration welds. Figure 4.2 shows a typical unreinforced design detail for a beam-to-column connection used in SMF systems prior to the 1994 Northridge earthquake. Common practice prior to the Northridge earthquake was to either bolt or weld the web to the column shear plate, and to weld the beam flanges to the column flange using a complete joint penetration groove weld. Historically, designers have assumed that beam shear is transferred to the column by the beam web connection and the moment is transferred through the beam flanges. However, recent studies by Lee (1997) indicate that this is not the actual case.

In the design of SMF connections, the engineer must set objectives for both load and deformation capacities. Usually, the load capacity requirement is based on the plastic moment of the beam. The connection must be strong enough to develop the strength of the beam, thus reducing the risk of brittle failure in the connection. Inelastic

deformation capacity is required to assure ductility in predetermined locations when subjected to large deformation demands.

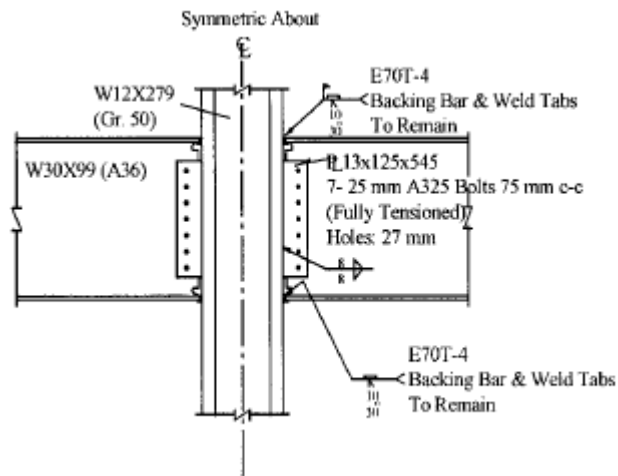


Figure 4.2 Typical unreinforced connection detail [14]

After the problem encountered in Northridge earthquake, a common philosophy has been accepted as to design the connection to remain nominally elastic and force the inelastic deformation of the frame to occur in a small portion of the beam which is a bit away from the connection [10]. This approach may be named as “capacity design” approach. The plastic moment and associated shear forces of the beam is based on probable strengths of materials. These maximum values are then considered as the design values for the connection. The connection of the beam to the column flange is executed by using these values.

The connections which have been derived after the Northridge earthquake mostly locate the possible plastic hinges away from the connection by reinforcing a portion of the beam close to the column [15]. Increase in stiffness in this area of the frame surely forces the inelastic deformations to occur away from the connection, just adjacent to the reinforced part of the beam. This scheme seems reasonable to apply, but material costs are evident due to the need for using extra steel plates to strengthen the ends of the beam. Moreover, welds also produce extra costs.

4.3 Background of Reduced Beam Section (RBS)

An alternative way of forcing the inelastic deformation to occur away from the connection zone is referred to as a “reduced beam section” connection, shortly RBS. This type of connection has the idea of selective removal of beam flange material adjacent to the beam-to-column connection, mostly from both top and bottom flanges, so as to reduce the moment capacity by reducing the cross sectional area at a predetermined location in the beam. Many different shapes of cutouts are possible; such as constant cut, tapered cut, radius cut and other types. Radius-cut reduced beam section is shown in Figure 4.3 .

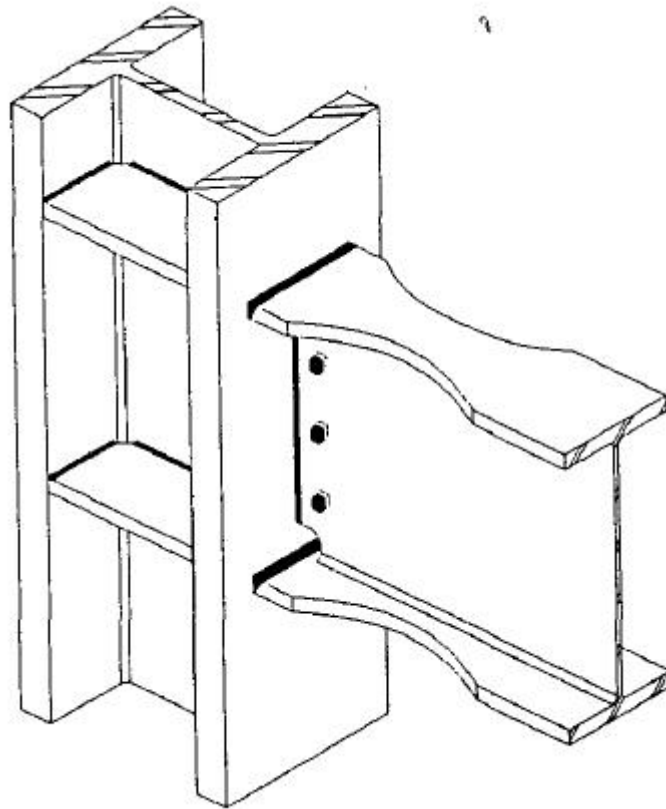


Figure 4.3 Radius-cut reduced beam section

The shape, size and location of the RBS all have an effect on the global behaviour of the subassembly. Different shapes have been tested for many kind of geometrical properties. Structural tests have been performed to investigate the behaviour of straight cut, tapered cut and radius cut reduced beam sections. Figure 4.4 shows the tapered-cut reduced beam section.

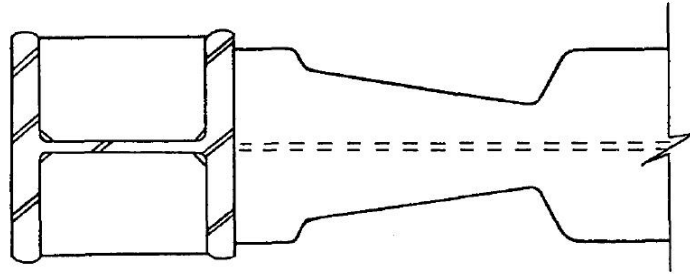


Figure 4.4 Tapered-cut reduced beam section.

The RBS surely forces yielding and hinge formation to occur within the reduced section of the beam and limits the moment that may develop at the face of the column. The reduction in demands on the beam flange groove welds and the surrounding base metal alters the possibility of fractures that occur in this region of the system.

The focusing of inelastic deformation that occurs in a small portion of the beam brings together the ability to take minimal protective measures at the connection of beam-to-column connection. The moment, which is smaller than the value for previously designed frames, reduces the stress demand on the welds, and also offers advantage in satisfying “strong column - weak beam” requirements. The need for doubler plates is also altered.

As mentioned before, many post-Northridge type beam-to-column connections end up with high costs for added material, extra welds and workmanship. The reduced beam section, however, does not bring such additional costs regarding extra plates or welds.

4.4 Initial Research

A significant amount of research and testing on RBS moment connections has already been completed in USA and additional work is underway. The experiments considered investigations on key parameters such as member sizes and material strengths, connection details, plastic rotation.

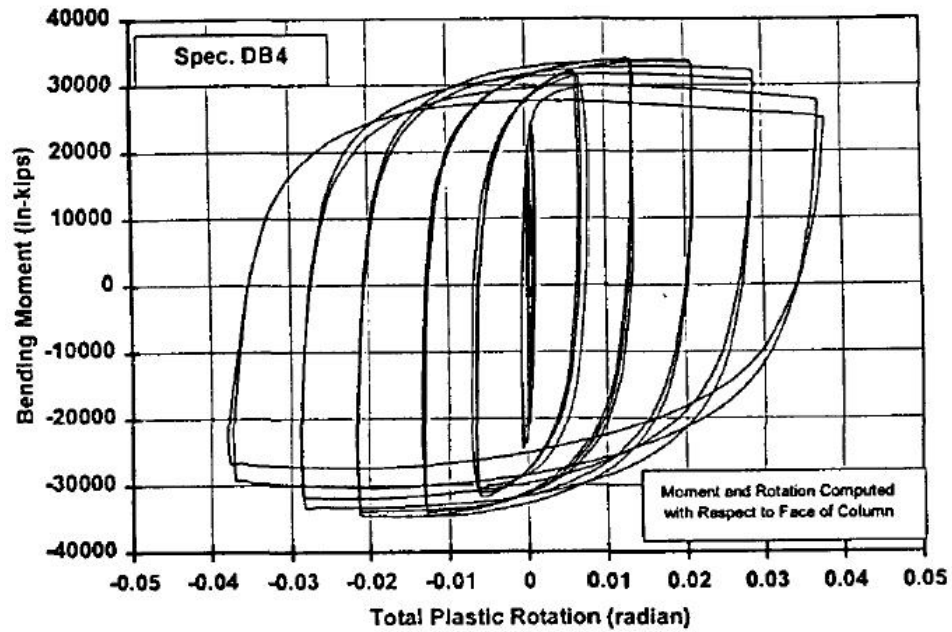


Figure 4.6 Response of the test specimen.

4.5 Closure

In this chapter , the concept of reduced beam section has been explained in detail. Whereas the aforementioned investigations have dealt with the inelastic behaviour of this type of connections, the modification in the geometry of the connection also modifies the rigidity of the structural steel framing system. One possible change in frame characteristics is the overall (or global) rigidity. The reduction of cross sectional area in small portions of the beam affects the stiffness of the system. In the next chapter, the main part of this study will be presented, with the emphasis on the effect of radius cut reduced beam section on lateral stiffness of steel frames, which will clearly demonstrate the effect on elastic behaviour.

5. FINITE ELEMENT MODELING OF RBS CONNECTIONS

5.1 Overview

Finite element analyses are used to gain better insight into the behaviour of welded beam-to-column connections and in particular to evaluate the effect of various connection modifications prior to experimental testing. They can also be used to give direction to the testing program by estimating the force requirements needed to reach given displacement limits. Although some information regarding the location of regions of high stress (stress concentration) can be obtained from a linear elastic analysis, substantial redistribution of stress occurs once the material yields. Therefore, it is preferable to use a nonlinear analysis procedure which considers both material and geometrical nonlinearities since at ultimate load, the connection specimen should experience strong material nonlinearity and possible geometric nonlinearity as well. However, for the sake of the purpose of this study, only linear static analyses have been performed.

Detailed and linear elastic finite element analyses are conducted throughout this study. The results are then compared with each other and change in performance and behaviour for various subassemblages having different geometric properties are evaluated.

5.2 The Finite Element Software and Its Capabilities

Solving real life problems has become much easier with the aid of general finite element analysis softwares. At this point, the software that has been used throughout this study should be mentioned.

The general purpose program, NISA, developed and marketed by Engineering Mechanics Research Corporation (EMRC) was well suitable for the analyses, for it

has many different modules which enables the user to solve many complex problems about many different subjects. The NISA family of programs consists of separate modules for dealing with linear and nonlinear elastic analysis with any kind of nonlinearity (e.g. material, geometrical, or both) , eigenvalue analysis, steady state and nonlinear transient heat transfer analysis, etc. The software can also handle problems of fluid mechanics, shape optimization, fatigue and fracture analyses, etc. Only the NISA STATIC module which is well able to deal with linear and nonlinear static analyses has been used for the investigations [16].

5.3 Macro Evaluation for Finite Element Models

Because the investigations require many finite element models of different but similar beams with built-up sections, setting up a macro for quick creation of the models would be very effective and time-saving. Therefore, the macro which may be found in Appendix was written. Running the macro shortened the consumed time for creating the models considerably. For a typical frame, it took only a couple of minutes to reach the state of being able to run the analysis.

5.4 Elastic Behaviour – Modeling and Results

As mentioned before in the preceding chapters, typical effect of the reduced beam section on the elastic behaviour of steel moment resisting frame may be assessed by the change in stiffness properties of the system.

5.4.1 The framework element stiffness matrix

Throughout the analyses, a frame element which is straight, prismatic and symmetrical about its both principal cross-sectional axes and which has 12 degrees of freedom (6 degrees of freedom for each end) has been taken into consideration. For such kind of section, the shear center coincides with the centroid. This general frame element has the widest field of application for solving even complex structural problems in structural design offices. Here, the internal forces resulting from unit axial deformation, unit flexural deformation in both principal axes and unit twisting deformation are obtained. Transverse shearing deformations and warping effects are

omitted throughout the analyses by controlling the end support conditions, namely restraints. Everyday design problems always force the designers to realize the flexural, axial and shear behaviour of structural systems, whereas seldom appears the need for detailed coverage of behaviour including warping. The borders for the analyses have been drawn according to this condition.

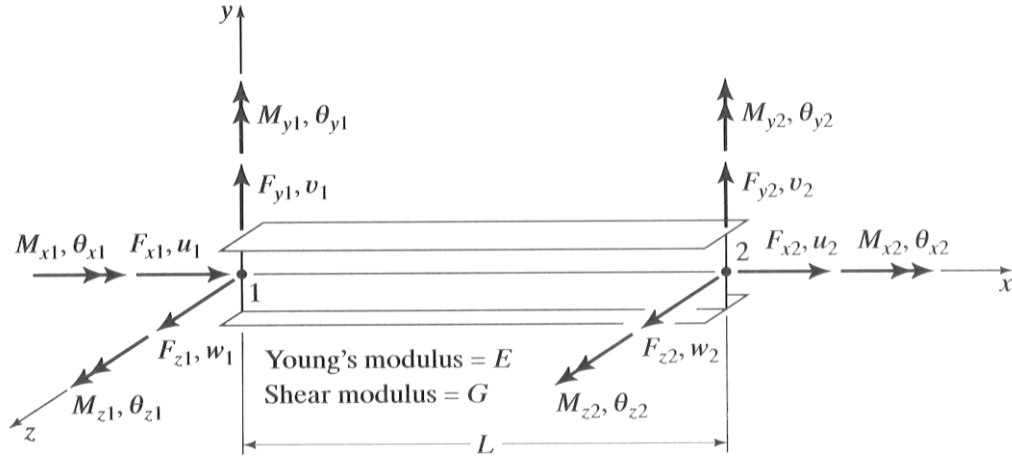


Figure 5. 1 Axis and internal force convention for the frame element

The type of member that has been studied is illustrated in Figure 5.1 with its orientation with respect to a local coordinate system and the nomenclature that has been used in the analyses. As can be seen in the figure, local x axis coincides with the centroidal axis of the element and local z axis represents the major principal axis (strong axis) whereas local y axis represents the minor principal axis (weak axis). Positive directions are also shown in the figure [17].

Aforementioned restrictions have provided a general purpose 12-degree-of-freedom frame element which has three dimensional stiffness characteristics. These effects are uncoupled, which means that a particular force vector causes a displacement only in the same vector direction. By using this property, the following four cases have been studied by means of finite element analyses:

1. Axial force
2. Bending about major principal axis
3. Bending about minor principal axis

4. Pure torsion

These four cases have been created for the built-up I shape structural steel section. Determination of the dimensions for this section has been explained in the following section.

The complete stiffness matrix for this frame element is presented in the following section, accompanied with the numerical values.

5.4.2 Determination of the cross sectional properties

For this purpose, an imaginary but typical design problem has been established. Supposing that a beam spanning 6 meters is to be designed, appropriate structural steel cross section with built-up shape is selected.

The beam is assumed to be fixed at either ends and is intended to carry a uniform distributed load of 25 kN/m. General layout of the structural system is illustrated in Figure 5.2 .

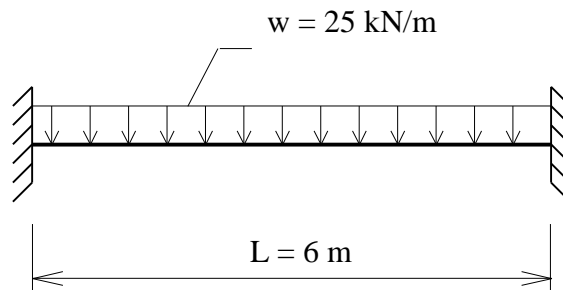


Figure 5.2 – General layout of the beam

Design procedure for this type of beams can readily be found in elementary level strength of materials or structural steel design textbooks, therefore these calculations are omitted here. After elementary calculations for determining the necessary cross section, the dimensions have been found as in Figure 5.3 .

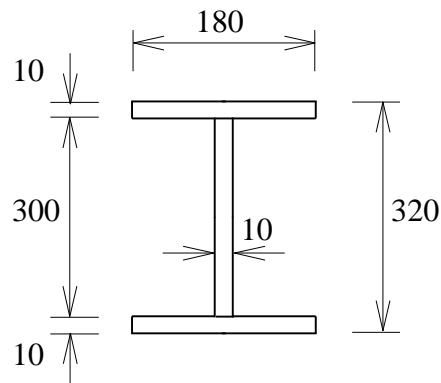


Figure 5. 3 Beam cross section (dimensions in millimeters)

As the cross section is determined, the stiffness equation for the frame element can be constructed easily. This equation uses some geometrical properties of this cross section. These properties are shown in tabulated form below.

Table 5.1 Geometrical properties of the cross section

A	66 cm ²
I _z	10902 cm ⁴
I _y	975 cm ⁴
J	22 cm ⁴

In the table above A is the cross-sectional area, I_z is the moment of inertia with respect to the major principal axis (here, local z axis) , I_y is the moment of inertia with respect to the minor principal axis (here, local y axis) and J is the torsional constant.

5.4.3 Formation of the general stiffness matrix

The stiffness equation for the bisymmetrical frame element having 12-degree-of-freedom is shown on the following page as equation (5.1) . This equation shows the coefficients of the stiffness equation symbolically, whereas Equation (5.2) has the numerical values assigned to them. The span length (L) for this case is 6 meters.

In the equation, the symbol ψ denotes Poisson's ratio for the material of the frame element. For steel, this ratio is 0.3.

$$\left\{ \begin{array}{c} F_{x1} \\ F_{y1} \\ F_{z1} \\ M_{x1} \\ M_{y1} \\ M_{z1} \end{array} \right\} = \left[\begin{array}{cccccc|cccccc} \frac{EA}{L} & 0 & 0 & 0 & 0 & 0 & -\frac{EA}{L} & 0 & 0 & 0 & 0 & 0 \\ 0 & \frac{12EI_z}{L^3} & 0 & 0 & 0 & \frac{6EI_z}{L^2} & 0 & -\frac{12EI_z}{L^3} & 0 & 0 & 0 & \frac{6EI_z}{L^2} \\ 0 & 0 & \frac{12EI_y}{L^3} & 0 & -\frac{6EI_y}{L^2} & 0 & 0 & 0 & -\frac{12EI_y}{L^3} & 0 & \frac{6EI_y}{L^2} & 0 \\ 0 & 0 & 0 & \frac{EJ}{2(1+\nu)L} & 0 & 0 & 0 & 0 & 0 & -\frac{EJ}{2(1+\nu)L} & 0 & 0 \\ 0 & 0 & -\frac{6EI_y}{L^2} & 0 & \frac{4EI_y}{L} & 0 & 0 & 0 & \frac{6EI_y}{L^2} & 0 & \frac{2EI_y}{L} & 0 \\ 0 & \frac{6EI_z}{L^2} & 0 & 0 & 0 & \frac{4EI_z}{L} & 0 & -\frac{6EI_z}{L^2} & 0 & 0 & 0 & \frac{2EI_z}{L} \end{array} \right. \\ \hline \left\{ \begin{array}{c} F_{x2} \\ F_{y2} \\ F_{z2} \\ M_{x2} \\ M_{y2} \\ M_{z2} \end{array} \right\} = \left[\begin{array}{cccccc|cccccc} -\frac{EA}{L} & 0 & 0 & 0 & 0 & 0 & \frac{EA}{L} & 0 & 0 & 0 & 0 & 0 \\ 0 & -\frac{12EI_z}{L^3} & 0 & 0 & 0 & -\frac{6EI_z}{L^2} & 0 & \frac{12EI_z}{L^3} & 0 & 0 & 0 & -\frac{6EI_z}{L^2} \\ 0 & 0 & -\frac{12EI_y}{L^3} & 0 & \frac{6EI_y}{L^2} & 0 & 0 & 0 & \frac{12EI_y}{L^3} & 0 & \frac{6EI_y}{L^2} & 0 \\ 0 & 0 & 0 & -\frac{EJ}{2(1+\nu)L} & 0 & 0 & 0 & 0 & 0 & \frac{EJ}{2(1+\nu)L} & 0 & 0 \\ 0 & 0 & \frac{6EI_y}{L^2} & 0 & \frac{2EI_y}{L} & 0 & 0 & 0 & \frac{6EI_y}{L^2} & 0 & \frac{4EI_y}{L} & 0 \\ 0 & \frac{6EI_z}{L^2} & 0 & 0 & 0 & \frac{2EI_z}{L} & 0 & -\frac{6EI_z}{L^2} & 0 & 0 & 0 & \frac{4EI_z}{L} \end{array} \right. \quad (5.1)$$

$$\begin{Bmatrix} F_{x1} \\ F_{y1} \\ F_{z1} \\ M_{x1} \\ M_{y1} \\ M_{z1} \end{Bmatrix} = \begin{bmatrix} 231000 & 0 & 0 & 0 & 0 & 0 & -231000 & 0 & 0 & 0 & 0 & 0 \\ 0 & 1271.9 & 0 & 0 & 0 & 3815700 & 0 & -1271.9 & 0 & 0 & 0 & 3815700 \\ 0 & 0 & 113.75 & 0 & -341250 & 0 & 0 & 0 & -113.75 & 0 & -341250 & 0 \\ 0 & 0 & 0 & 2961538.462 & 0 & 0 & 0 & 0 & 0 & -2961538.462 & 0 & 0 \\ 0 & 0 & -341250 & 0 & 1365000000 & 0 & 0 & 0 & 341250 & 0 & 682500000 & 0 \\ 0 & 3815700 & 0 & 0 & 0 & 15262800000 & 0 & 3815700 & 0 & 0 & 0 & 7631400000 \end{bmatrix} \begin{Bmatrix} u_1 \\ v_1 \\ w_1 \\ \theta_{x1} \\ \theta_{y1} \\ \theta_{z1} \end{Bmatrix} \\
 \begin{Bmatrix} F_{x2} \\ F_{y2} \\ F_{z2} \\ M_{x2} \\ M_{y2} \\ M_{z2} \end{Bmatrix} = \begin{bmatrix} -231000 & 0 & 0 & 0 & 0 & 0 & 231000 & 0 & 0 & 0 & 0 & 0 \\ 0 & -1271.9 & 0 & 0 & 0 & 3815700 & 0 & 1271.9 & 0 & 0 & 0 & -3815700 \\ 0 & 0 & -113.75 & 0 & 341250 & 0 & 0 & 0 & 113.75 & 0 & 341250 & 0 \\ 0 & 0 & 0 & -2961538.462 & 0 & 0 & 0 & 0 & 0 & 2961538.462 & 0 & 0 \\ 0 & 0 & -341250 & 0 & 682500000 & 0 & 0 & 0 & 341250 & 0 & 1365000000 & 0 \\ 0 & 3815700 & 0 & 0 & 0 & 7631400000 & 0 & -3815700 & 0 & 0 & 0 & 15262800000 \end{bmatrix} \begin{Bmatrix} u_2 \\ v_2 \\ w_2 \\ \theta_{x2} \\ \theta_{y2} \\ \theta_{z2} \end{Bmatrix} \quad (5.2)$$

The stiffness values which are shown on the equations above arise from mainly four different and uncoupled deformation patterns. These deformations were indicated before as axial deformation, bending deformations about two principal axes and the twisting deformation, also known as torsion. Reducing the beam flange width over a predetermined portion of the beam will surely affect the stiffness terms of the frame element having twelve degrees of freedom.

This effect on the stiffness properties of the frame element can easily be characterized using one modification coefficient for each of the principal deformations which were outlined above. These modification factors clearly show the effect of the reduced beam section on the stiffness characteristics of the frame element.

These modification factors imply that, instead of using the conventional and characteristic geometrical properties (such as moments of inertia, cross sectional area or torsional constant), the design engineer should take the effect of reduced beam width into account. Most structural design softwares have an option to modify the frame element properties by defining some modification factors. The factors derived in this study are believed to find area of application in everyday office work.

5.4.4 Finite element analyses – analysis procedure and results

The stiffness method of analysis (also known as the displacement method) uses the displacements at the end nodes of an element to obtain the internal forces within that element. The stiffness coefficients are then used with these end deformations to obtain the internal forces. The stiffness matrix is formed in this way.

First, unit displacements (linear and angular) were introduced at the left end of the element model as support displacements within that point. Since for small displacements the axial force effects, bending about each axis and torsion are uncoupled, the influence coefficients relating these effects are zero. Therefore, deformation in one direction does not produce any internal force in any direction other than itself. Knowing this, the corresponding support reactions at those nodes were noted.

In the tables below, the stiffness solutions which are calculated by evaluation of the coefficients in (5.2) and which are obtained from static finite element analyses are summarized for each of the four principal deformation cases.

During the finite element analyses, the first four terms on the diagonal of the complete stiffness matrix are evaluated. The other coefficients were not directly investigated, because they easily evolve from the equilibrium equations of a frame element, therefore the modification factors for the first four terms on the diagonal are valid for the other related terms of the stiffness matrix.

From this point on, the modification factors arising from the comparison of the two stiffness values for the same unit displacement will be denoted as k_x , where the subscript x will denote the related geometrical property of the cross section. The symbol k_A will denote the modification of the axial stiffness in terms of area, whereas for the change in bending characteristics k_{I_z} and k_{I_y} will be used for modifying bending about the major and minor principal cross sectional axes in terms of moments of inertia according to these axes, respectively. The factor k_J will be used for the change in torsional behaviour characteristic in terms of the torsional constant.

At first, unit axial displacement was applied at one end as support movement. This movement consequently causes reaction in the direction of the centroidal axis. For four different beam flange cutout ratios, the support reactions are given in Table 5.2 below. Note that the theoretical value was found as 231000 N/mm. The values of k_A are found by dividing the results of the finite element analyses

Table 5.2 Axial stiffness coefficients (L=6 m)

Flange cutout percentage	Unit reaction at node due to unit displacement ($u_x=1$ mm)	Ratio k_A
0%	234778 N/mm	1.016
20%	233098 N/mm	1.009
40%	230609 N/mm	0.998
50%	229004 N/mm	0.990

The results demonstrate that reduction in flange width does not considerably effect the axial rigidity of the frame element. The area of the gross section without any reduction in flange width can be relied on during the design.

Table 5.3 shows the stiffness coefficients due to strong axis bending. The theoretical approach yields the value as 1271.9 N/mm. In this case, the value of I_z is obtained by applying unit rotation at node 1 of the model. The support reaction then indicates the strong axis bending stiffness. Comparing them with the theoretical values has provided the modification factors for strong axis bending.

Table 5.3 Strong axis bending stiffness coefficients (L=6 m)

Flange cutout percentage	Unit reaction at node due to unit displacement ($u_y=1$ mm)	Ratio k_{Iz}
0%	1260.23 N/mm	0.991
20%	1227.00 N/mm	0.965
40%	1179.77 N/mm	0.928
50%	1148.50 N/mm	0.903

The values in Table 5.3 show that 50 % flange reduction yields about 10 % decrease in strong axis bending stiffness. This is very important, because this stiffness coefficient is the main factor in controlling lateral stiffness of structural frames. Structural sections are often placed with their strong axes perpendicular to the plane of large spans to be able to use the strong axis bending rigidity of sections for limiting drift values of structural systems.

Next, the modification factors for weak axis bending are investigated. Theoretical value is 113.75 N/mm. The results are shown in Table 5.4 .

Table 5.4 Weak axis bending stiffness coefficients (L=6 m)

Flange cutout percentage	Unit reaction at node due to unit displacement ($u_z=1$ mm)	Ratio k_{ly}
0%	113.559 N/mm	0.998
20%	100.980 N/mm	0.888
40%	79.3844 N/mm	0.698
50%	64.4895 N/mm	0.567

As was already anticipated, reduction of flange width has considerable effect on the weak axis bending stiffness. Although the reduced beam section scheme is not intended for weak axis bending deformations, the possibility of lateral buckling of beams is closely related with weak axis bending stiffness of beams. Therefore, it was considered necessary to include the effect on weak axis stiffness in this study.

As fourth case, unit twisting deformation (unit torsion) was applied to the beam with reduced beam sections. The required external force to maintain the deflected shape of the frame element due to unit twisting deformation is calculated as 2961538.462 Nmm/mm. The results are shown in Table 5.5 .

Table 5.5 Torsional stiffness coefficients (L=6 m)

Flange cutout percentage	Unit reaction at node due to unit twisting rotation ($\theta_x=1$ rad)	Ratio k_J
0%	2849940 N/mm	0.962 (1.000)
20%	2309220 N/mm	0.780 (0.810)
40%	2299440 N/mm	0.776 (0.807)
50%	2293300 N/mm	0.774 (0.805)

Although the ratios for 0% reduced (uncut) beam sections in earlier stiffness investigations were close to unity, this is not the case for torsional deformation. The reason for this appears to be the ability of the finite element model to account for shear deformations within the cross section. Theoretical values are computed based on assumptions, such as neglect of transverse shear stresses. The difference is believed to arise from this point. However, the ratio is still very close to unity, only

4% difference from the theoretical value is observed. The ratios are shown also by taking this 4% difference into account by normalizing the ratios for various cutout percentages with respect to the first ratio for the beam without any reduced beam section. These values are stated in parantheses.

From the table above, the reduction in torsional rigidity can be observed as about 20-25% . This shows that reducing beam width results in a considerable reduction in torsional rigidity.

So far, stiffness coefficients and relevant modification factors have been presented for a span length of six meters. It was also stated that the quick creation of models by the help of a macro was a straightforward and easy process. The modification factors (called “ratios” in the tables) were assumed to be independent from the span lenth, L . These factors were taken as effective only upon the sectional geometrical properties of the cross section, such as moment of inertia. It is now the time to investigate whether this assumption is still valid for a larger span.

For this purpose, a new but similar set of models were created only by altering the input variable “span length (L)” within the macros. The same approach was taken for determining the modification factors for a span length of 10 meters. A new cross section was not determined, for the main purpose of this further investigation is the effect of longer span length on the stiffness degradation. Similar tables show the analysis results and the ratios (modification factors) for a span length of 10 meters.

The theoretical value for axial stiffness of this 10-meter-long frame is 138600 N/mm. The results of the finite element analyses are compared with this value.

Table 5.6 Axial stiffness coefficients ($L=10$ m)

Flange cutout percentage	Unit reaction at node due to unit displacement ($u_x=1$ mm)	Ratio k_A
0%	140800 N/mm	1.016
20%	140194 N/mm	1.012
40%	139290 N/mm	1.005
50%	138703 N/mm	1.001

The ratios are as expected, like the ones for a span of six meters.

Table 5.7 shows the stiffness coefficients due to strong axis bending. The theoretical approach yields the value as 274.7304 N/mm.

Table 5.7 Strong axis bending stiffness coefficients (L=10 m)

Flange cutout percentage	Unit reaction at node due to unit displacement ($u_y=1$ mm)	Ratio k_{Iz}
0%	277.675 N/mm	1.011
20%	272.958 N/mm	0.994
40%	265.791 N/mm	0.967
50%	261.044 N/mm	0.950

As observed in the table above, increasing the span length has decreased the rate of degradation of strong axis bending stiffness.

In Table 5.8 , stiffness coefficients due to weak axis bending are shown. The theoretical value for this condition yields 24.57 N/mm. The ratio of the results with respect to this value are shown in the table below.

Table 5.8 Weak axis bending stiffness coefficients (L=10 m)

Flange cutout percentage	Unit reaction at node due to unit displacement ($u_z=1$ mm)	Ratio k_{Iy}
0%	24.5575 N/mm	0.999
20%	22.7364 N/mm	0.925
40%	19.2333 N/mm	0.783
50%	16.4893 N/mm	0.671

Table 5.9 shows the resultant forces due to unit torsional rotation. The division of the results to the theoretical value of 1776923 Nmm/mm are tabulated.

Table 5.9 Torsional stiffness coefficients (L=10 m)

Flange cutout percentage	Unit reaction at node due to unit twisting rotation ($\theta_x=1$ rad)	Ratio k_J
0%	1525310 N/mm	0.858 (1.000)
20%	1522230 N/mm	0.857 (0.998)
40%	1518010 N/mm	0.854 (0.995)
50%	1515360 N/mm	0.853 (0.993)

It is seen that increasing the span length has decreased the rate of torsional stiffness degradation in a considerable manner.

In order to combine the results for two different span lengths, the modification factors for each displacement cases will now be tabulated. The first table below shows the change of axial stiffness modification factor with respect to beam cutout percentage and span length.

Table 5.10 Variation of k_A with cutout percentage and span length

Flange cutout percentage	k_A modification factor	
	for L = 6m	for L = 10m
0%	1.016	1.016
20%	1.009	1.012
40%	0.998	1.005
50%	0.990	1.001

From Table 5.10 , it may be concluded that increasing the span length does not affect the modification factor k_A .

Modification factors for bending stiffness about strong axis is shown in Table 5.11 .

Table 5.11 Variation of k_{Iz} with cutout percentage and span length

Flange cutout percentage	k_{Iz} modification factor	
	for L = 6m	for L = 10m
0%	0.991	1.011
20%	0.965	0.994
40%	0.928	0.967
50%	0.903	0.950

Table 5.11 demonstrates that stiffness degradation decreases as the span length increases. This conclusion is believed to be useful during structural design, as using reduced beam section for a longer span will have a less adverse effect on the bending about major principal, in other words strong axis.

Table 5.12 Variation of k_{Iy} with cutout percentage and span length

Flange cutout percentage	k_{Iy} modification factor	
	for L = 6m	for L = 10m
0%	0.998	0.999
20%	0.888	0.925
40%	0.698	0.783
50%	0.567	0.671

Table 5.12 above shows the variation of the modification factor k_{Iy} with the changing values of cutout depth percentage and span length. Again, stiffness degradation is much slower in the beam with longer span length.

Table 5.13 Variation of k_J with cutout percentage and span length

Flange cutout percentage	k_J modification factor	
	for L = 6m	for L = 10m
0%	0.962 (1.000)	0.858 (1.000)
20%	0.780 (0.810)	0.857 (0.998)
40%	0.776 (0.807)	0.854 (0.995)
50%	0.774 (0.805)	0.853 (0.993)

The results for torsional stiffness change are very similar to the ones before. As the span increases, the degradation slows down considerably.

5.5 Closure

In this chapter, detailed finite element analyses for the investigation of the effect of reduced beam section on stiffness characteristics have been conducted. The results show that this structural fuse effects the frame element behaviour

6. SUMMARY AND CONCLUSIONS

The experience that has been gained aftermath this earthquake provides many innovative solutions of earthquake resistant structural design. With regard to the common damage to welded beam-to-column connections, steel moment resisting frames with reduced beam sections may be considered as structural systems that are ready for future earthquakes.

Throughout this study, the effect of this reduced beam section on the stiffness characteristics of the frame element has been investigated by means of finite element analyses. Many results are obtained regarding the stiffness changes about axial deformation, bending and twisting deformation (torsion). Modification factors for the corresponding geometrical properties are evaluated for two different span lengths.

Degradation of stiffness with respect to bending about the major principal axis of the cross section was found as nearly 10 %. These coefficients may be used in everyday design works reliably by modifying the member geometrical properties. The interesting point was that, as the span length increased, the reduction in corresponding stiffness parameters decreased. This shows that reduction in flange widths of beams which span over a large space may also be applied.

The conclusion is that, this technique which inserts a “structural fuse” in the framing system is very promising for future applications of structural steel framing system design. For example, retrofitting a damaged frame without using any additional plates etc. decreases the cost for post-earthquake rehabilitation of these damaged buildings.

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APPENDIX

MACRO,BEAM,A1/A2/B1/B2/C1/C2/LB/DB/BF1/BF2/TF1/TF2/TW

**

** PART 1 - BEAM

**

SET,LABA,OFF

VEW,ISO

**

** *****

** *** GRID DEFINITION ***

** *****

**

** Grids will be located to define the geometry
** of the beam by using the input variables above.

**

GRD,ADD,1,0 / 0 / 0

GRD,ADD,2,(0.50*(A1+A2)) / 0 / 0

GRD,ADD,3,(0.50*(A1+A2+B1+B2)) / 0 / 0

GRD,ADD,4,(A1+A2+B1+B2) / 0 / 0

GRD,ADD,5,((1.25)*(A1+B1+A2+B2)) / 0 / 0

GRD,ADD,6,(LB) / 0 / 0

**

GRD,ADD,7,0 / 0 / (0.5*(DB-TF1))

GRD,ADD,8,(A1) / 0 / (0.5*(DB-TF1))

GRD,ADD,9,(A1+(B1/2)) / 0 / (0.5*(DB-TF1))

GRD,ADD,10,(A1+B1) / 0 / (0.5*(DB-TF1))

GRD,ADD,11,(2*(A1+B1)) / 0 / (0.5*(DB-TF1))

GRD,ADD,12,(2*(A1+B1)+0.5*(A1+B1)) / 0 / (0.5*(DB-TF1))

GRD,ADD,13,(LB) / 0 / (0.5*(DB-TF1))

**

GRD,ADD,14,0 / (-0.50*BF1) / (0.50*(DB-TF1))

GRD,ADD,15,(A1) / (-0.50*BF1) / (0.50*(DB-TF1))

GRD,ADD,16,(A1+0.50*B1) / -((0.5*BF1)-C1) / (0.50*(DB-TF1))

GRD,ADD,17,(A1+B1) / (-0.50*BF1) / (0.50*(DB-TF1))

GRD,ADD,18,(2*(A1+B1)) / (-0.50*BF1) / (0.50*(DB-TF1))

GRD,ADD,19,((2*(A1+B1)) + 0.5*(A1+B1)) / (-0.50*BF1) / (0.50*(DB-TF1))

GRD,ADD,20,(LB) / (-0.50*BF1) / (0.50*(DB-TF1))

**

GRD,ADD,21,0 / 0 / (-0.5*(DB-TF2))

GRD,ADD,22,(A2) / 0 / (-0.5*(DB-TF2))

GRD,ADD,23,(A2+B2) / 0 / (-0.5*(DB-TF2))

GRD,ADD,24,(2*(A2+B2)) / 0 / (-0.5*(DB-TF2))

GRD,ADD,25,((2.5)*(A2+B2)) / 0 / (-0.5*(DB-TF2))

GRD,ADD,26,(LB) / 0 / (-0.5*(DB-TF2))

```

**
GRD,ADD,27,0 / (-0.50*BF2) / (-0.50*(DB-TF2))
GRD,ADD,28,(A2) / (-0.50*BF2) / (-0.50*(DB-TF2))
GRD,ADD,29,(A2+(B2/2)) / ((-0.50*BF2)+C2) / (-0.50*(DB-TF2))
GRD,ADD,30,(A2+B2) / (-0.50*BF2) / (-0.50*(DB-TF2))
GRD,ADD,31,(2*(A2+B2)) / (-0.50*BF2) / (-0.50*(DB-TF2))
GRD,ADD,32,((2.5)*(A2+B2)) / (-0.50*BF2) / (-0.50*(DB-TF2))
GRD,ADD,33,(LB) / (-0.50*BF2) / (-0.50*(DB-TF2))
**
GRD,ADD,34,(A2+(B2/2)) / 0 / (-0.5*(DB-TF2))
**
**
** LINE DEFINITION
**
** Lines should be generated along the curved edge of the
** reduced beam section so as to define patches easily.
**
LIN,NGD,15/16/17
LIN,2GD,8,9
LIN,2GD,9,10
LIN,NGD,28/29/30
LIN,2GD,22,34
LIN,2GD,34,23
**
**
** PATCH DEFINITION
**
** 4 grids or 2 lines are used to define
** the geometry of the patches.
**
**
** TOP FLANGE
**
PAT,NLIN,0/0
PAT,NEC,8
PAT,4GD,14/15/8/7
PAT,2LN,1,3
PAT,2LN,2,4
PAT,4GD,17/18/11/10
PAT,4GD,18/19/12/11
PAT,4GD,19/20/13/12
**
**
** WEB
**
PAT,NEC,2
PAT,4GD,1/2/8/7
PAT,4GD,2/3/10/8
PAT,4GD,3/4/11/10
PAT,4GD,5/6/13/12

```

PAT,4GD,21/22/2/1
 PAT,4GD,22/23/3/2
 PAT,4GD,23/24/4/3
 PAT,4GD,25/26/6/5
 **
 **
 ** BOTTOM FLANGE
 **
 PAT,NEC,7
 PAT,4GD,27/28/22/21
 PAT,2LN,5,7
 PAT,2LN,6,8
 PAT,4GD,30/31/24/23
 PAT,4GD,31/32/25/24
 PAT,4GD,32/33/26/25
 PAT,4GD,24/25/12/11
 **
 ** TOTAL 21 PATCHES FOR THE BEAM
 **
 **
 **
 ** FINITE ELEMENT GENERATION
 **
 ** All finite elements will be
 ** 8-node & 2nd order shell elements
 **
 ** BEAM TOP FLANGE
 **
 FAM,QUA,1,,,2/2/2/2,20/2/1/1,1/1/1/1
 FAM,QUA,2,,,2/2/2/2,20/2/1/1
 FAM,QUA,3,,,2/2/2/2,20/2/1/1
 FAM,QUA,4,,,6/2/6/2,20/2/1/1
 FAM,QUA,5,,,1/1/2/2,20/2/1/1
 FAM,QUA,6,,,8/1/8/1,20/2/1/1,2/1/2/1
 **
 ** BEAM WEB
 **
 FAM,QUA,7,,,2/2/2/2, 20/2/1/2
 FAM,QUA,8,,,4/2/4/2, 20/2/1/2
 FAM,QUA,9,,,6/2/6/2, 20/2/1/2
 FAM,QUA,10,,,8/1/8/1, 20/2/1/2, 2/1/2/1
 FAM,QUA,11,,,2/2/2/2, 20/2/1/2
 FAM,QUA,12,,,4/2/4/2, 20/2/1/2
 FAM,QUA,13,,,6/2/6/2, 20/2/1/2
 FAM,QUA,14,,,8/1/8/1, 20/2/1/2, 2/1/2/1
 FAM,QUA,21,,,2/2/2/4/1, 20/2/1/2
 **
 ** BEAM BOTTOM FLANGE
 **
 FAM,QUA,15,,,2/2/2/2, 20/2/1/3, 1/1/1/1

FAM,QUA,16,,,2/2/2/2, 20/2/1/3
FAM,QUA,17,,,2/2/2/2, 20/2/1/3
FAM,QUA,18,,,6/2/6/2, 20/2/1/3
FAM,QUA,19,,,1/1/2/2, 20/2/1/3
FAM,QUA,20,,,8/1/8/1, 20/2/1/3, 2/1/2/1
**
**
**
SET,LABA,OFF
SET,ELAB,PRO
ELE,PAI,PRO
VEW,HID,ON
PLO,ACT
**
**
** PROPERTY SPECIFICATION
**
PROP,ADD,1,8M(TF1)
PROP,ADD,2,8M(TW)
PROP,ADD,3,8M(TF2)
**
**
ENDMACRO

VITA

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